Appendix D
Miter Gate Design Example

The opening in this example is 36 ft wide by 11.125 ft high.

Load Cases:

In accordance with EM 1110-2-2502, consideration was given to the following cases applicable to inland floodwalls.

Case I1, Design Flood Loading. Gate is mitered; water on the unprotected side is at the design flood elevation; water is at or below sill on protected side. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as Case I1 except that water level is to top of gate on unprotected side. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

Case I3, Earthquake Loading. Water is at usual level (nonflood condition) on unprotected side; earthquake-induced forces are acting. (Note: This case is applicable to support structures only.)

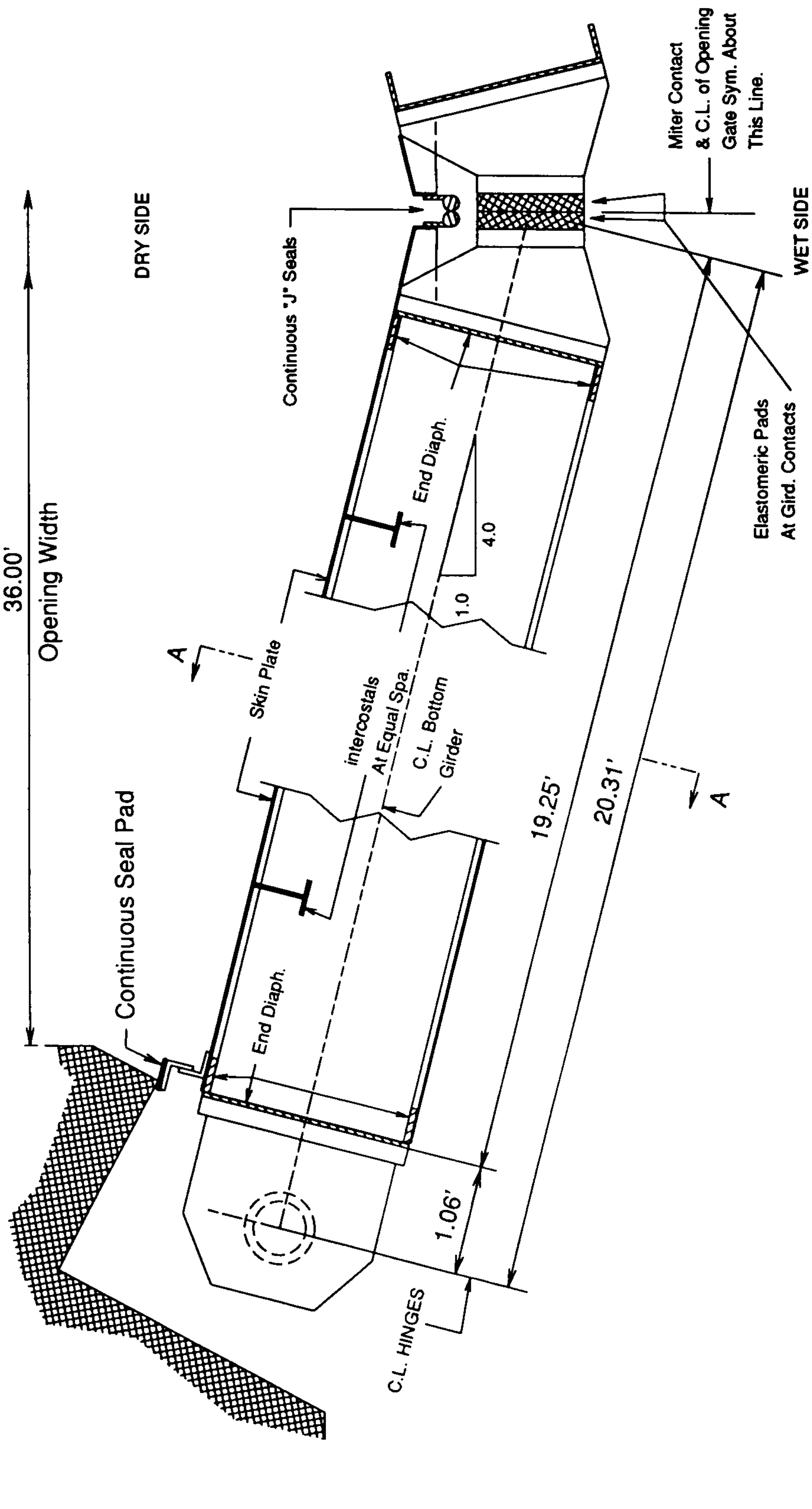
Case I4, Short-Duration Loading. Gate is either open or mitered or in between and is subjected to construction and/or wind loads. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

In this example, cases I1 and I3 are not significant and skin plate, intercostals, and girders are designed for Case I2. Case I4 is applicable to the design of diagonals and latching devices.

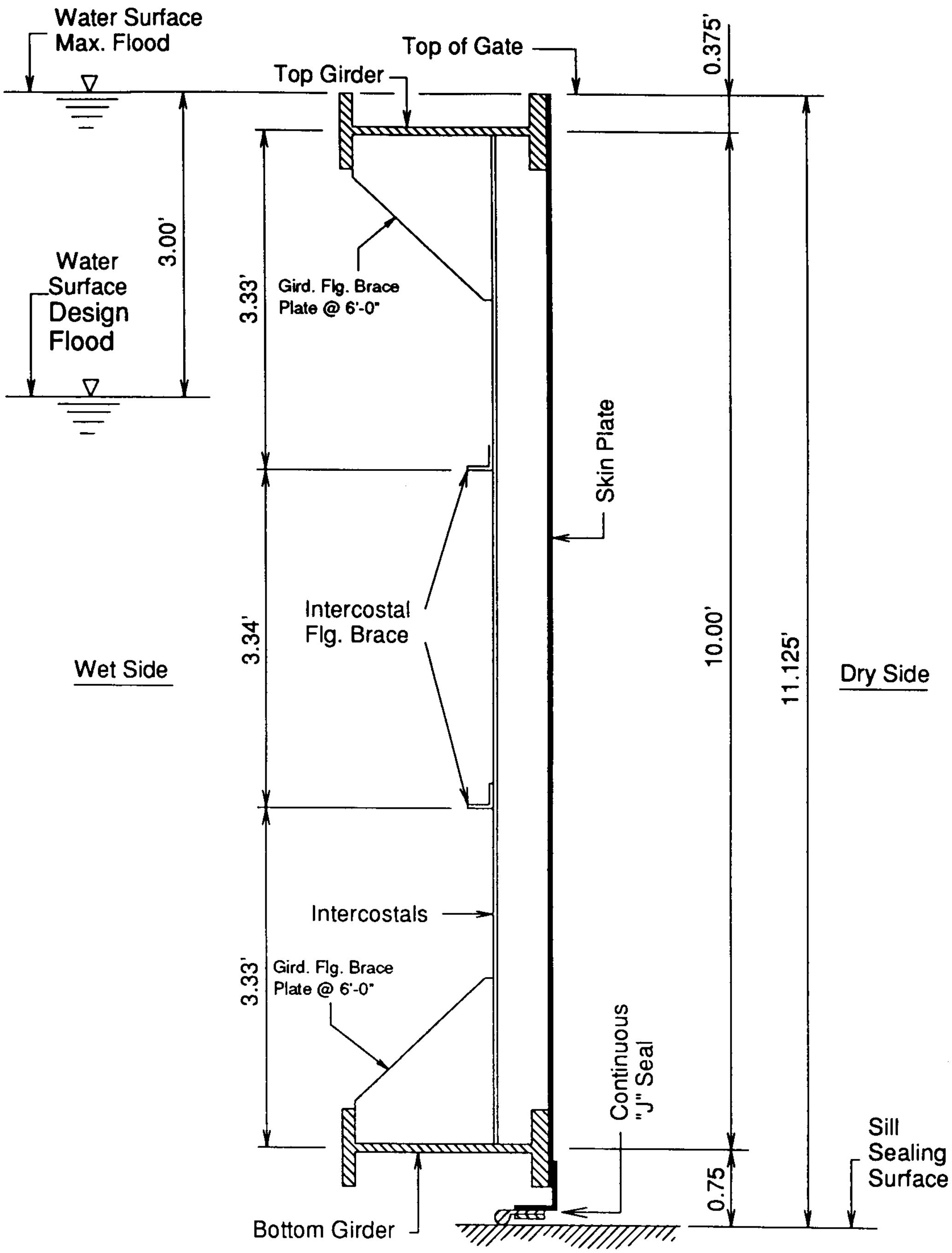
The skin plate is designed as a fixed end member spanning between intercostals. The hydrostatic pressure 6 ft above the flange of the bottom girder is used as a uniform load. In order for the design to meet the limitations of the flat plate theory, deflection is limited to 0.4 of plate thickness.

The intercostals are designed as simple beams spanning between girders.

Girders are designed as elements of a three-hinged arch. They are designed for thrusts and moments induced by diagonal tensions as well as for hydrostatic pressure.



1/2 HORIZONTAL SECTION THRU GATE


Water Surface
Max. Flood
Top of Gate
Top Girder
0.375'
3.00'
Water
Surface
Design
Flood
3.33'
Gird. Flg. Brace
Plate @ 6'-0"
Skin Plate
Intercostal
Flg. Brace
3.34'
Wet Side
10.00'
11.125'
Dry Side
Intercostals
3.33'
Gird. Flg. Brace
Plate @ 6'-0"
Continuous
"J" Seal
Sill
Sealing
Surface
0.75
Bottom Girder

SECTION A–A

Skin Plate Design; Load Case I2:

Assuming 9" wide girder flanges, the hydrostatic pressure for skin plate design is:

$p=0.0625(11.125-0.75-0.375-.5)=0.5983ksf$

$p=0.004124ksi$, b=intercostal spacing=24"

$M=pb^2/12=0.004124(24)^2/12=0.1980k\text{-}in$

$F_b=1.11(0.75F_y) = 1.11x0.75x36 = 30ksi$

$t_{min\text{-}stress}=(pb^2/2F_b)^{1/2}$

$t_{min\text{-}stress}=[0.004124(24)^2/(2x30)]^{1/2}=0.1990"$

Defl.$=pb^4/384EI$; E=29000; $I=t^3/12$; Defl.=0.4t

$0.4t=12pb^4/384Et^3$, $t^4=pb^4/12.8E$

$t_{min\text{-}defl.}=[0.004124(24)^4/(12.8x29000)]^{1/4}$

$t_{min\text{-}defl.}=\underline{0.2462"}$

USE: 1/4" Skin Plate.

Intercostal Design; Load Case I2:

Eq. & Table No's. In Parentheses Are From AISC (1989).
Load, shear, and moment diagrams for intercostals are shown on page D-5.

M=4.27x2'=8.54k-ft=102.48k-in

The trial section for intercostals is shown on page D-6.

$S=5.47in^3$, L_b=40"(dist. between flg. braces)

$b_f/2t_f=3.96/(2x0.21)=9.43$

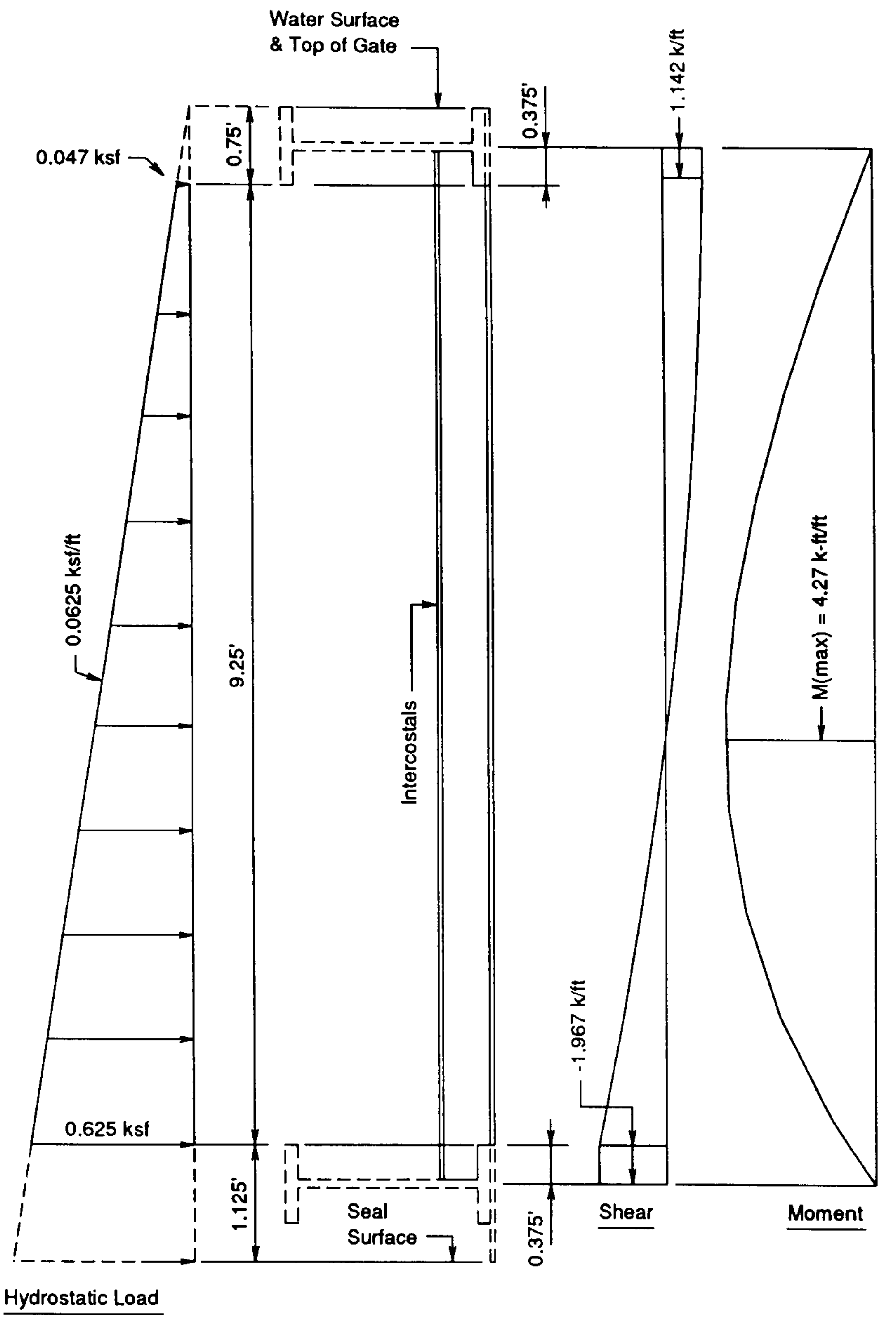
$65/(F_y)^{1/2}=65/6=10.83>9.43$, compact (Table B5.1)

$L_c=76b_f/(F_y)^{1/2}=76(3.96)/6 = 50.16">40"$ (F1-2)

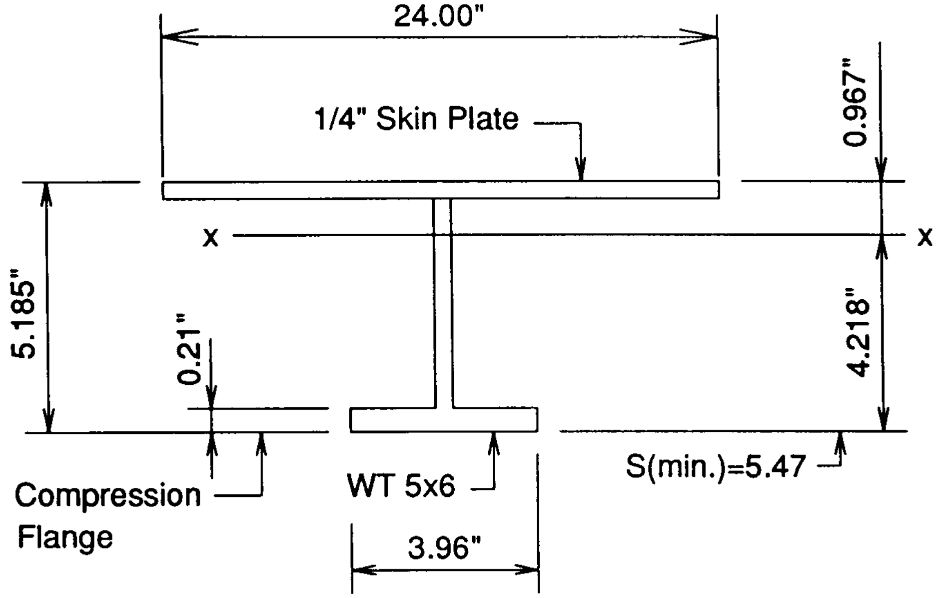
$F_b=1.11(2/3)F_y=1.11(2/3)(36) = 26.67ksi$

$f_b=M/S=102.48/5.47=18.73ksi<F_b=26.67ksi$

USE: WT 5x6 for Intercostals.



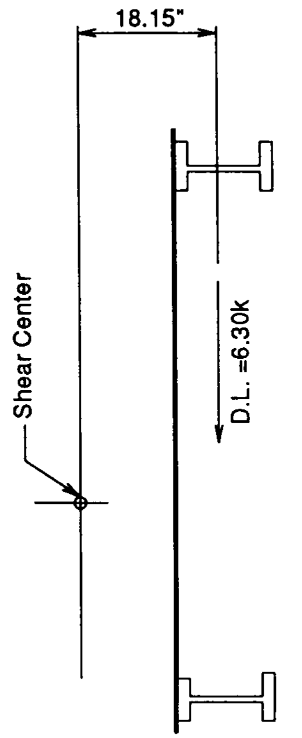
LOAD, SHEAR, & MOMENT DIAGRAMS
FOR
INTERCOSTALS



INTERCOSTAL SECTION

CALCULATION FOR DIAGONAL LOADS

Methods For Determining The Shear Center And The Torsional Stiffness Of The Gate Are Presented In EM 1110-2-2703.



End View

T=Torsion=6.30x18.15=114.35k-in

Z=Distance From Hinges To C.G. Of D.L.

Z=123"

TZ=14065k-in^2

A=Area Of Diagonal=0.79

A'=1/8 Of Sum Of Girder & Diaph. Areas

A' = 8.05

L=Length Of Diagonal=253.24"

Qo=Torsional Stiffness Of Gate Leaf=1222

t=Dist. From C.L. Of Skin To C.G. Of Diagonal

t =24.2"

Ro=2wt/vL=2x223x24.2/(243.72x253.24)=0.175

R=A' Ro/(A+A')

R=8.05x0.175/(0.79+8.05)

R=0.159

Q=RRoEAhv/L=Torsional Stiff. Diagonal

Q=0.159x0.175x29000x0.79x120x243.72/253.24=73620

QDp + QDn = TZ Where Dp is Prestress Delection For Positive Diagonal and Dn is Prestress Deflection For Negative Diagonal.

Positive Diagonal Extends From Top Girder At Hinge End To Bottom Girder At Miter End.

Let Dn = 0.40", QDn = 73620x0.40 = 29448

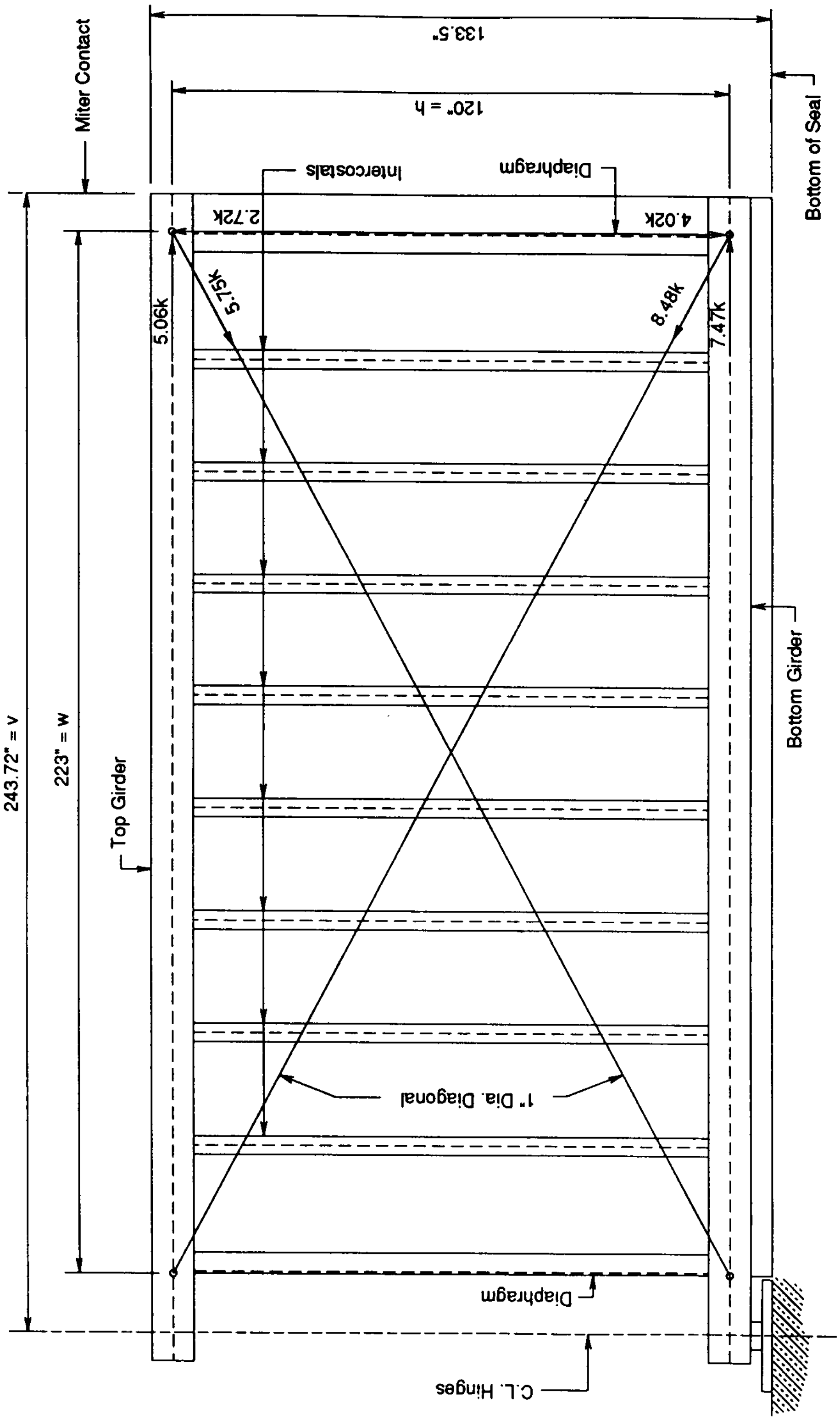
73620 Dp = 14065 + 29448 = 43513, Dp = 43513/73620 = 0.59"

Sp = RDpE/L = 0.159x0.59x29000/253.24 = 10.74ksi

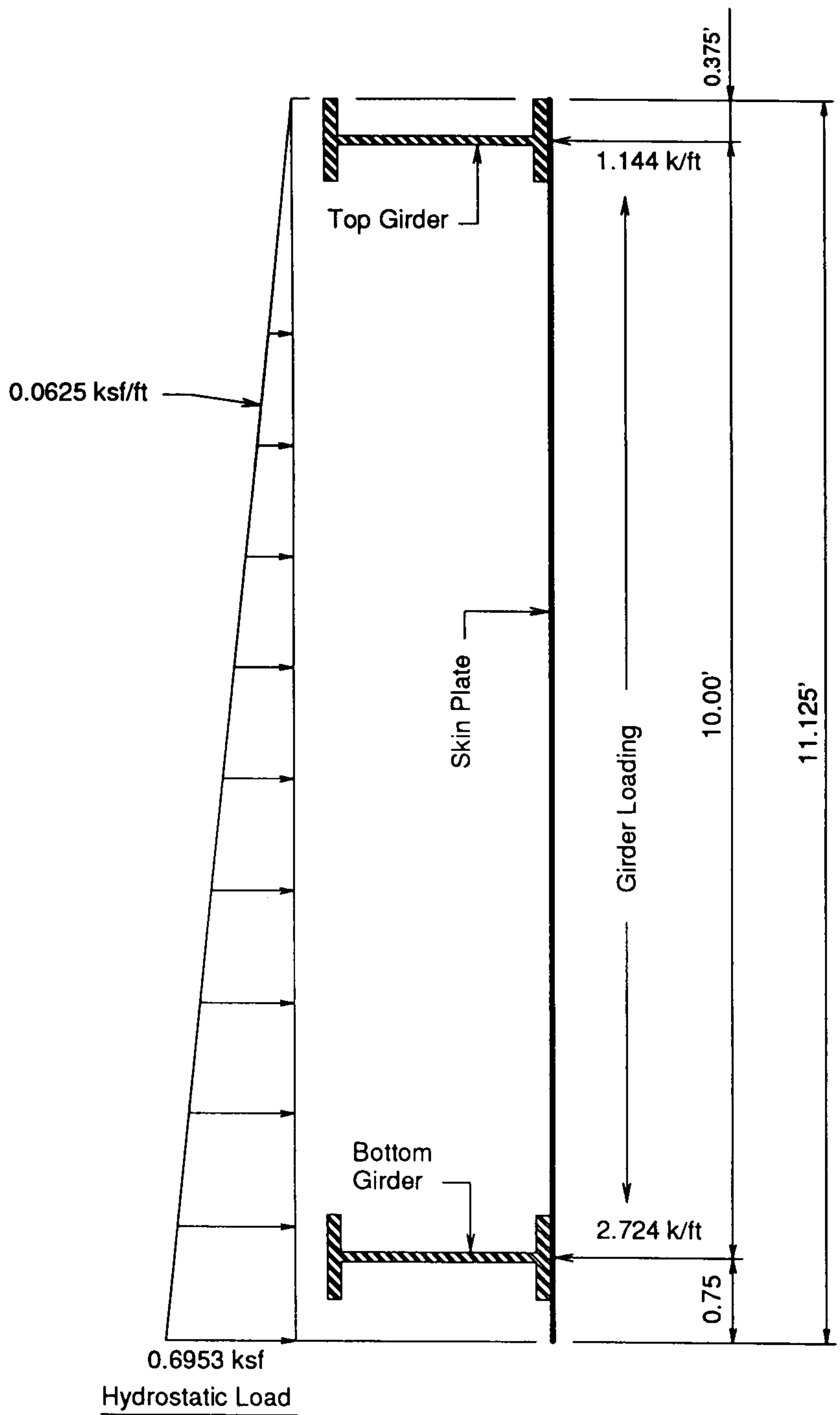
Sn = RDnE/L = 0.159x0.40x29000/253.24 = 7.28ksi

Tension In Positive Diagonal = 0.79x10.74 = 8.48k

Tension In Negative Diagonal = 0.79x7.28 = 5.75k

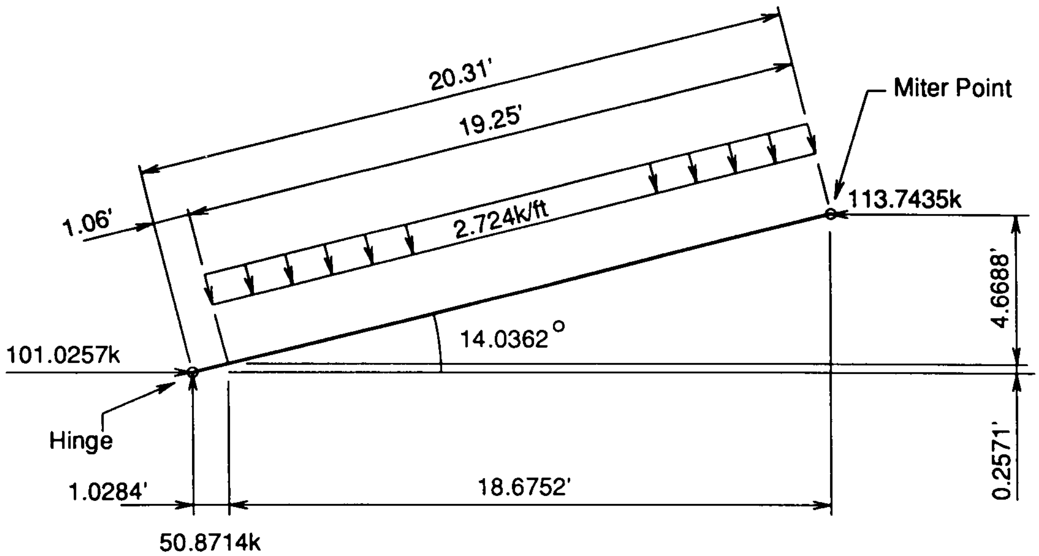


GATE LEAF ELEVATION
SHOWING DIAGONAL LOADS



GIRDER LOADING

HYDROSTATIC LOADING & REACTIONS FOR BOTTOM GIRDER:



Free Body Of Bottom Girder

Resolve Reactions At Hinge Into Components Normal And Parallel To Girder Centerline:

V = Normal Component, T = Parallel Component

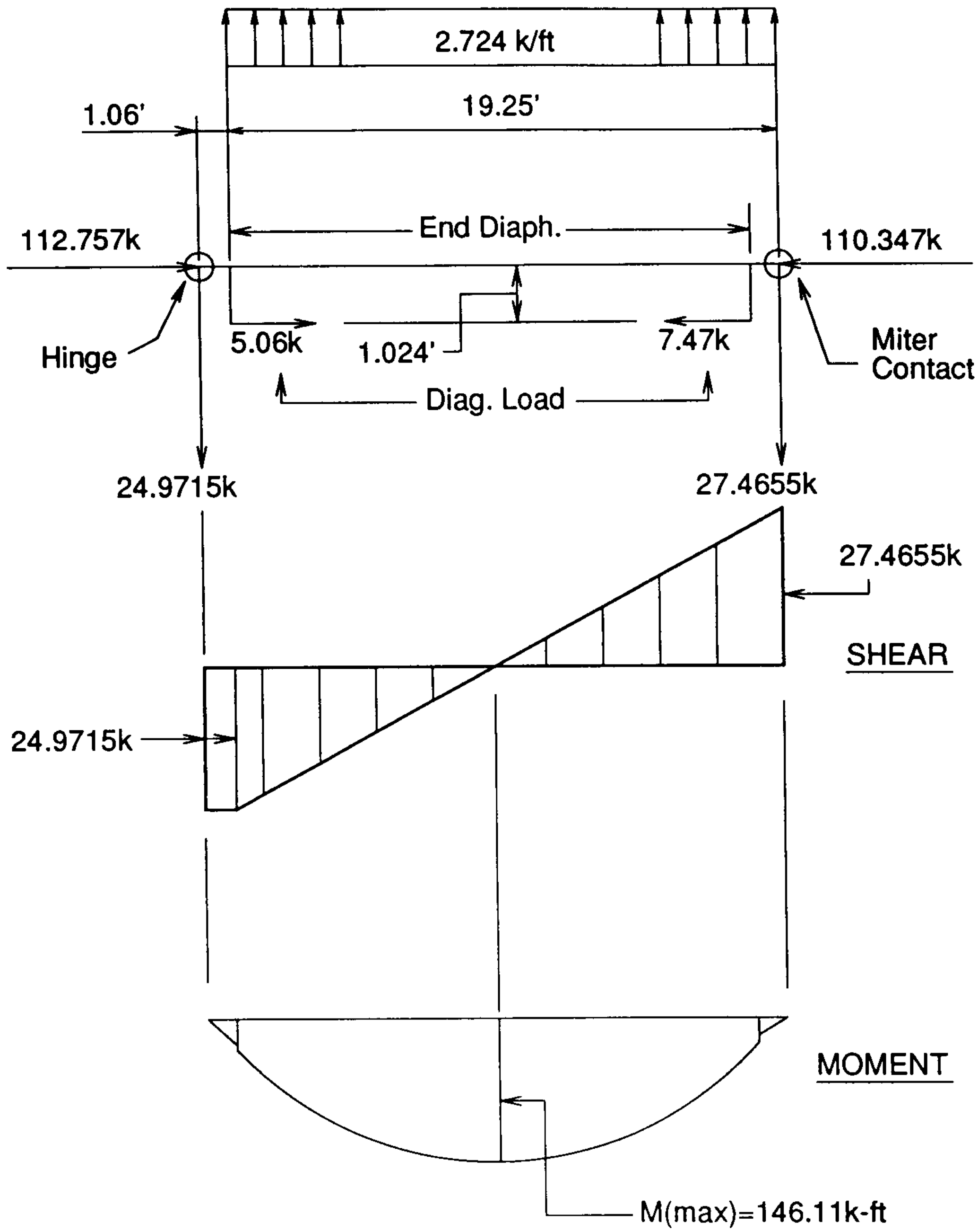
V = 50.8714xCos(14.0362) - 101.0257xSin(14.0362) =24.850k

T = 50.8714xSin(14.0362) + 101.0257xCos(14.0362) = 110.347k

Resolve Reaction At Miter Point Into Components Normal And Parallel To Girder Centerline:

V = 113.7435xSin(14.0362) = 27.587k

T = 113.7435xCos(14.0362) = 110.347k



LOAD, REACTIONS, SHEAR, & MOMENT
FOR
BOTTOM GIRDER

BOTTOM GIRDER DESIGN

Material: ASTM A36

Equation & Table Numbers In Parentheses Are From Spec. In AISC (1989).

Trial Section, W 24x55.

$A=16.2$, $S_x=114$, $r_x=9.11$, $r_y=1.34$, $r_T=1.68$, $d/Af=6.66$

$b_f=7"$, $t_f=0.505"$, $b_f/2t_f=6.9$, $d=23.57"$, $h=22.56"$

$t_w=0.395"$, $d/t_w=59.7$, $h/t=57.11$

$L_x=20.31'=244"$, $L_y=24"$, $L_b=72"$

$P=112.757+5.06=117.817k$, $M=146.11k\text{-}ft=1753.32k\text{-}in$

Allowable Stresses:

$65/(F_y)^{1/2}=65/6=10.83>6.9$ (Table B5.1)

$L_c=20000/(6.66x36)=83.42>L_b=72"$

$640/(F_y)^{1/2}=640/6=106.67>59.7$ (Table B5.1)

W 24x55 is compact for flexure, $F_b=1.11(2/3)F_y=26.67ksi$

$257/(F_y)^{1/2}=257/6=42.83<59.7$ (Table B5.1)

Web is a slender element under uniform compression.

$h_e=[253t_w/(f)^{1/2}][1\text{-}44.3t_w/h(f)^{1/2}]$ (A-B5-8)

$f_a=P/A=117.817/16.2=7.27ksi$, $f_b=M/S=1753.32/114=15.38ksi$

$f=7.27+15.38=22.65ksi$, $(f)^{1/2}=4.76$

$h_e=[253x0.395/4.76][1\text{-}44.3/(57.11x4.76)]=17.57"$

$A_e=16.2\text{-}(22.56\text{-}17.57)(0.395) = 14.23$

$Q=A_e/A=14.23/16.2=0.8784$

$L_x/r_x=244/9.11=27$, $L_y/r_y=24/1.34=18$

$F_a=1.11x20.15=22.39ksi$ (E2-1)

$F'_e=1.11x204.84=227.6ksi$ (From Sect. H1 of AISC (1989)

$F_a=QF_a=0.8784x22.39=19.67ksi$ (For Case I2)

Combined Stresses:

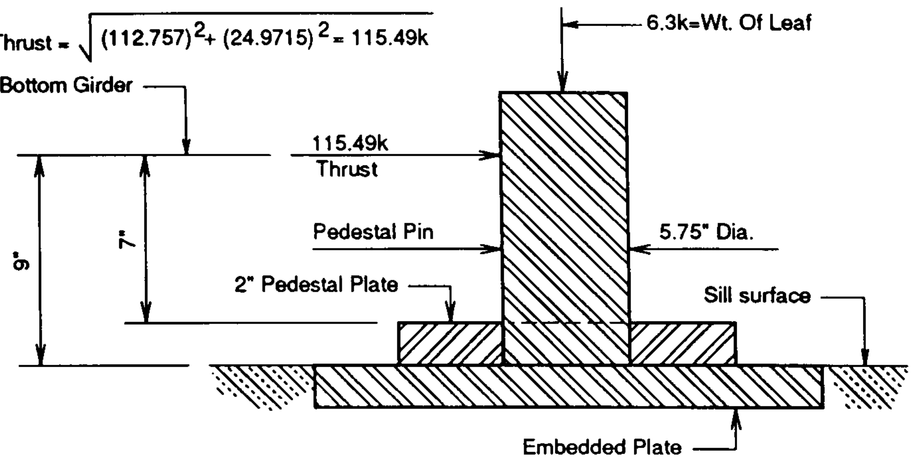
$f_a/F_a+Cmf_b/[1\text{-}f_a/F_e]F_b=<1.00'$ (H1-1)

$7.27/22.39 + 1x15.38/[(1\text{-}7.27/227.6)(26.67)]=0.92<1.00$

USE W24x55 — Also Use W 24x55 For Top Girder To Maintain 24" Depth So That Diagonals Lie In A Vertical Plane.

BOTTOM HINGE-PEDESTAL DESIGN

Result.Thrust = $\sqrt{(112.757)^2 + (24.9715)^2} = 115.49k$



Pin Mat'l.: ASTM A276

Section Thru Pedestal

Pedestal Pin

$F_y = 55$ ksi. $F_b = 1.11\ (0.75F_y) = 45.83$ksi

d = 5.75", A = 25.97 in^2, I = 53.66 in^4

S = 18.66 in^3, r = 1.44

L = 7", K = 2, KL/r =2x7/1.44 = 9.72

C_c=102 (Ref. 3.d.(1).)

FS = 5/3 + 3(KL/r)/(8xC_c) - [KL/rC_c]^3/8 = 1.70

Fa = [1-0.5(KL/Ccr)^2][Fy]/FS=32.17ksi (AISC 1989)

F_a = 1.11x32.17 = 35.74ksi

M = 115.49 x 7 = 808.43 k-in

f_a = 6.3/25.97 = 0.243 ksi

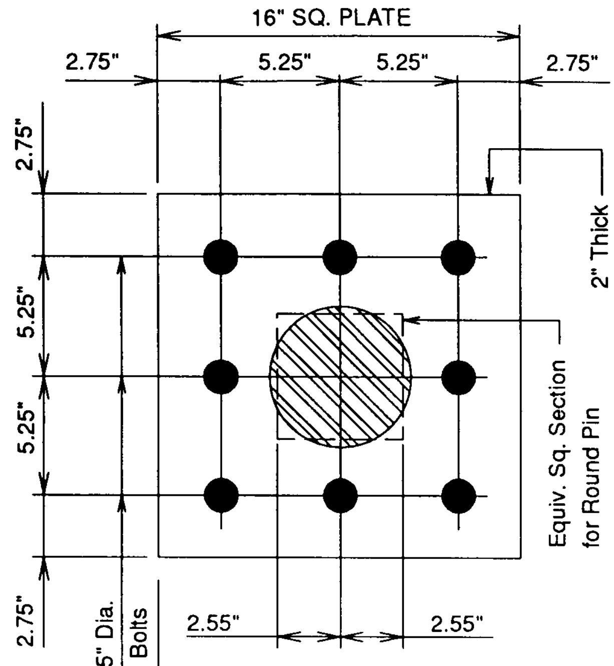
f_b = 808.43/18.66 = 43.32 ksi

$f_a/F_a + f_b/F_b$= 0.243/35.74+43.32/45.83=0.952<1.000

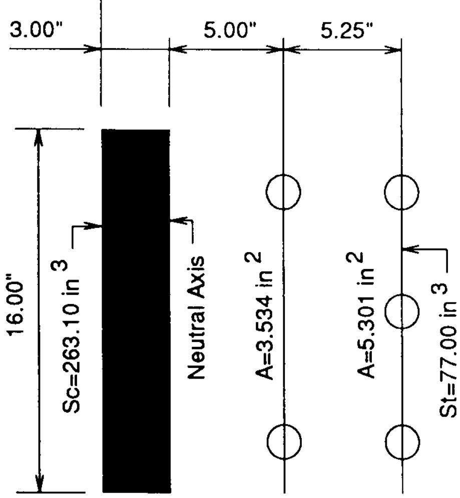
USE 5.75" Diameter Pin

PEDESTAL BASE PLATE

Material: Bolts F593, Alloy 316. Plate ASTM A276.



PLAN



PROPERTIES OF SECTION
AT
BOTTOM OF PLATE

BOLTS:

$V = 115.49k$

$M = 115.49x9 = 1039.41k\text{-}in$

$f_c = M/Sc$

$f_c = 1039.41/263.10 = 3.95ksi$

$f_t = M/St$

$f_t = 1039.41/77 = 13.50ksi$

Bolt Area At Thread = 1.41

f_t (at thread)=1.767x13.5/1.41=

= 16.92ksi

$F_y = 30ksi$

$F_t = 1.11x0.6x30 = 20ksi > 16.92ksi$

$f_v = 115.49/(8x1.41) = 10.24ksi$

$F_v = 1.11x0.4x30 = 13.33ksi > 10.24ksi$

USE 8-1.50" DIA. BOLTS

PLATE:

Moment in plate at edge of equiv. Sq. Pin.

$M = 0.5x3.95x3x4.45x16 = 421.86k\text{-}in$

$S = 16(2)^2/6 = 10.67$

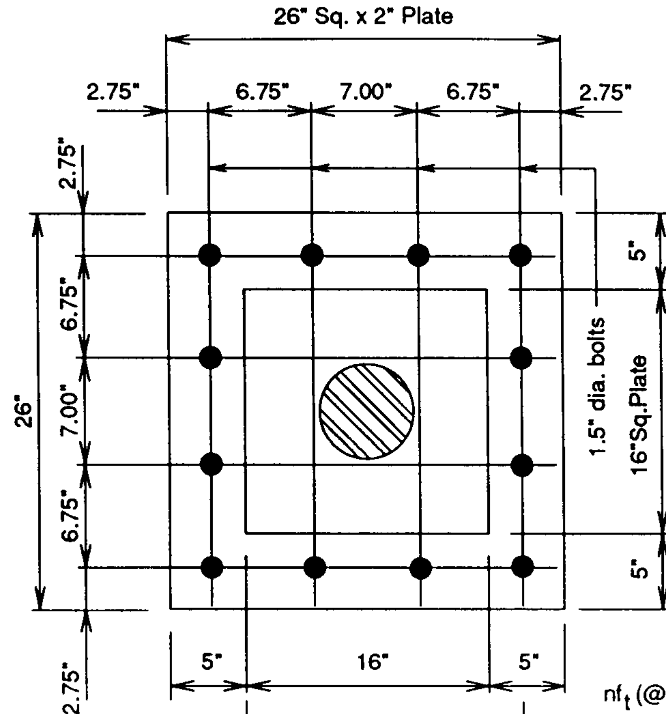
$f_b = M/S = 421.86/10.67 = 39.54ksi$

$F_y = 55ksi$

$F_b = 1.11X0.75X55 = 45.83ksi > 39.54ksi$

USE 16" SQUARE X 2" PLATE

EMBEDDED PLATE & BOLTS: (Bolt Mat'l. F593, Alloy 316. Plate A36)



PLAN

BOLTS:

Transformed Section, n = 10

$I_{N.A.}$=22519.21

Sc = 2338.44, St = 1653.39

V = 115.49k

M = 115.49x11 = 1270.39k-in

f_p= M/Sc = 0.543ksi

F_p= 1.11x0.35f'$_c$

F_p= 1.11x0.35x3 = 1.167ksi

$F_p > f_p$

f_t = M/St = 0.7684ksi

F_y = 30ksi

F_t= 1.11x0.6x30 = 20ksi

A(bolt)=1.767, A(@ th'ds.)=1.41

nf_t = 10x0.7684 =7.684ksi

nf_t (@th'ds)=1.767x7.684/1.41=9.63ksi

F_t > 9.63ksi

F_v= 1.11x0.4xF_y= 13.33ksi

f_v=115.49/(12x1.767)=5.45ksi<13.33ksi

USE 12-1.5" DIA. BOLTS

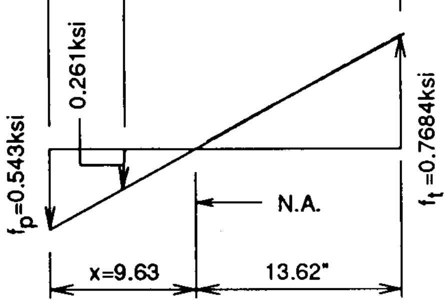
PLATE @ EDGE OF PED. PLATE:

M=0.5(0.543+0.261)(5)(2.79)(26)=

M = 145.80k-in , S=26(2)^2/6=17.33

f_b= M/S =8.41ksi

F_b= 1.11x0.67x36 = 26.67ksi > 8.41ksi



STRESS DIAGRAM

PLATE @ EDGE OF EQUIV.SQ. PIN:

M=0.5(0.543)(9.63)(7.24)(26)=

M = 492.16k-in

S(both plates)=10.67+17.33=28

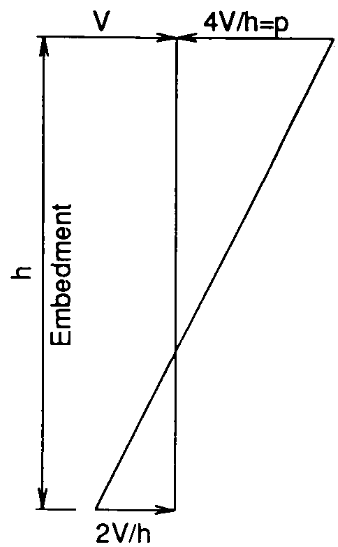
f_b=M/S = 17.58ksi < 26.67ksi

See Page D-16 For Bending Stress In Embedded Bolts & Bolt Bearing Stress On Concrete.

BOLTS IN CONCRETE

Using Maximum Allowable Concrete Bearing Stress Find the Minimum Required Bolt Embedment:

f'_c=3ksi (28 day concrete compressive strength)



h=Min. Embed.=4V/p

$F_p=0.35f'_c=1.05$ksi From Sect. J9 Of AISC (1989).

d = Bolt Dia. =1.5" F_p(for Case I2)=1.11x1.05=1.167ksi

$p = dF_p$ = 1.5x1.167 = 1.75k/in

Total Horizontal Load On Embedment=115.49k

Allowable Brg.Load On 2" Plate=1.167x26x2=60.68k

Shear Load On Embedded Bolts=115.49-60.68=54.81k

V=54.81/12=4.568k/bolt

h=4x4.568/1.75=10.44" (min req'd.)

Actual Embed.=17" > 10.44" o.k.

Calculate Flexural Stress In Bolt:

M=4Vh/27=4x4.568x10.44/27=7.065k-in

S = Bolt Sect. Mod. = 0.3313

$1.11(0.75F_y)$ = 1.11x0.75x30 = 25ksi = F_b

f_b= M/S =7.065/0.3313 = 21.33ksi < 25ksi o.k.

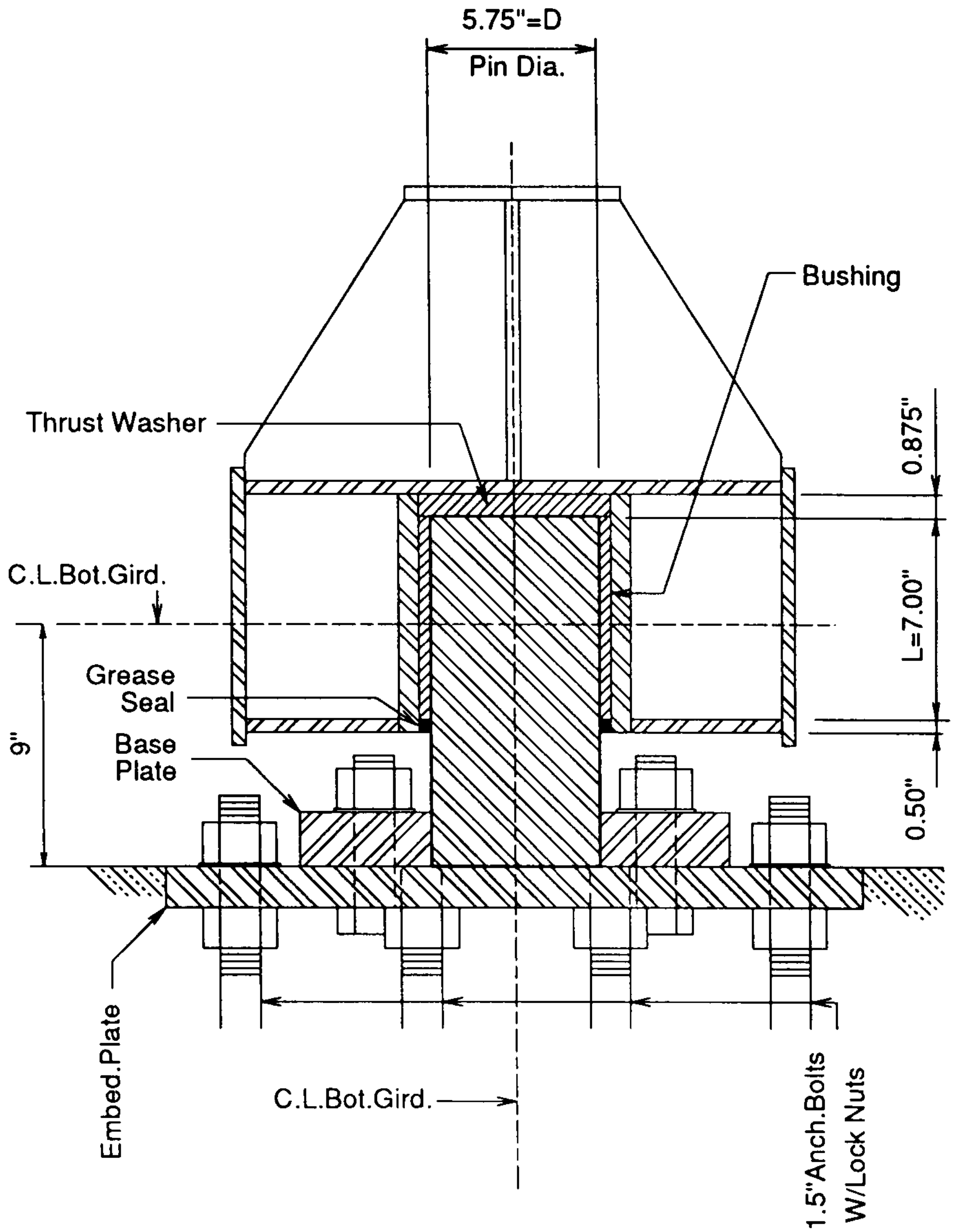
BUSHING DESIGN : (Mat'l. ASTM B22)

F_p=Max.Allowable Avg. Brg. Stress=1.11x3.00=3.33ksi

USE: L=7" , D=5.75" , P=115.49k , f_p=P/DL

f_p=115.49/(7x5.75)=2.869ksi<3.33ksi

See Page D-18 For Calculation Of Max. Bearing Stress.



SECTION THRU BOTTOM HINGE

CALCULATION FOR ACTUAL MAXIMUM BEARING PRESSURE-BOTTOM PIN

L = Bushing Length = 7", Nominal Pin Diameter = 5.75"

R1 = Min. Radius Of Pin = 2.8670"

R2 = Max. Inside Radius Of Bushing = 2.8715"

E1 = Modulus Of Elasticity Of Pin = 29000ksi

E2 = Modulus Of Elasticity Of Bushing = 15000ksi

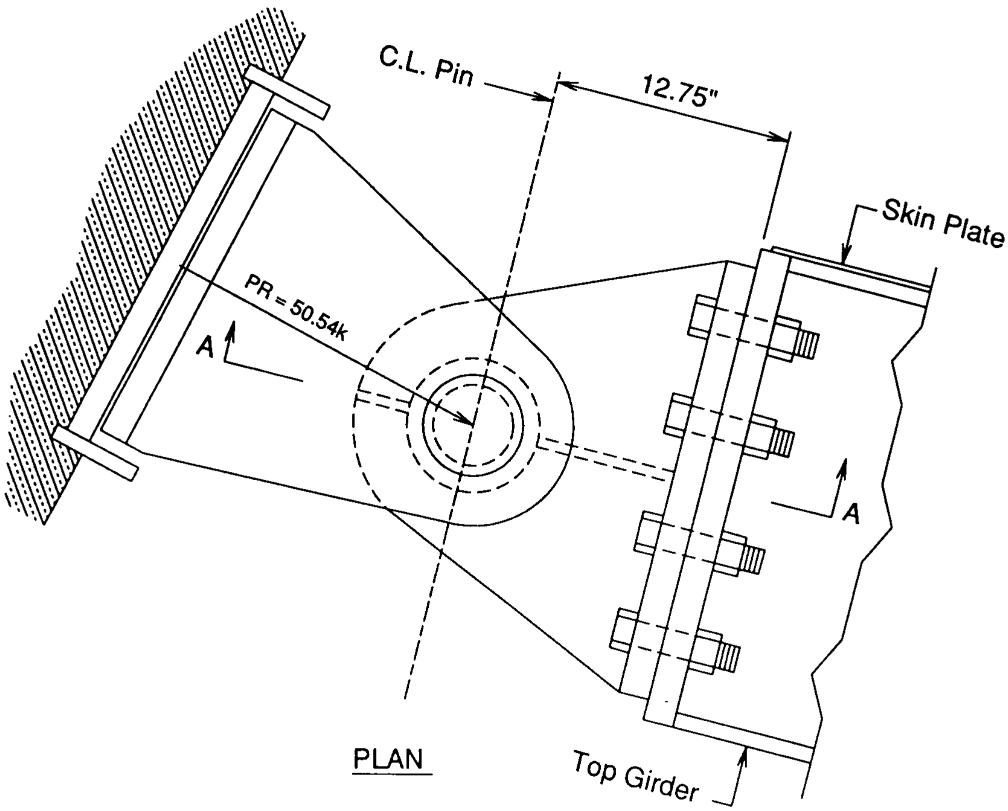
P1 = 115.49/7 = 16.50k/in

$$f_{p\ max.} = 0.591 \sqrt{\frac{P1\ E1\ E2\ (R2-R1)}{(E1+E2)(R1R2)}}$$

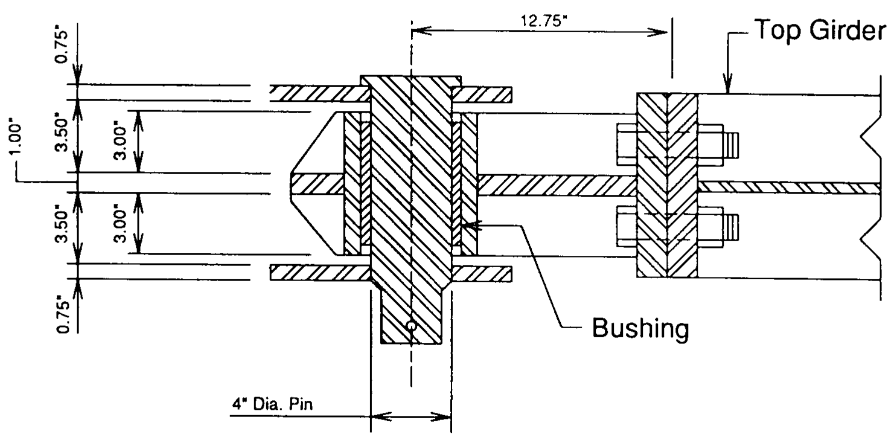
$$f_{p\ max.} = 0.591 \sqrt{\frac{16.5 \times 435 \times 10^{6} \times 0.0045}{44 \times 10^{3} \times 8.25843}} = 5.572\text{ksi}$$

A Maximum Bearing Stress Equal To Or Less Than The Yield Strength Of The Material Is Allowable.

TOP HINGE:



PLAN



SECTION A-A

TOP PIN (Mat'l. ASTM A276, Fy = 55ksi)

P_R = 50.54k, V = 50.54/2 = 25.27k (Dbl. Shear)

M = 50.54x8.75/4 = 110.56k-in

A = $3.1416(4)^2/4$ = 12.57, S = $3.1416(4)^3/32$ = 6.28

F_b=1.11(0.75F_y)=45.83ksi

F_V = 1.11(0.4F_y) = 24.44ksi

f_V= 25.27/12.57 = 2.01ksi < 24.44ksi

f_b= 110.56/6.26 = 17.61ksi < 45.83ksi

TOP BUSHING (Mat'l. ASTM B22)

P_R = 50.54k, Bushing Length = 6.00"

Inside Dia. = 4.00"

F_p= Avg. Allowable Brg. Stress =1.11x3.00 = 3.33ksi

f_p= 50.54/(4x6) = 2.11ksi < 3.33ksi

CHECK MAX. BEARING STRESS:

R1 = 1.997", R2 = 2.001"

E1 = 29000ksi, E2 = 15000ksi

P1 = 50.54/6 = 8.42k/in

$$(\text{Max})\ f_p = 0.591\sqrt{\frac{8.42 \times 29 \times 15 \times 10^6 (2.001-1.997)}{(29+15)(10^3)(2.001 \times 1.997)}} = 5.396\text{ksi}$$

A Maximum Bearing Stress Equal To Or Less Than The Yield Strength Of The Material Is Allowable.